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USE OF MATHEMATICAL MODELS AS AIDS TO DECISION MAKING IN WATER QUALITY CONTROL

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USE OF MATHEMATICAL MODELS AS AIDS TO DECISION MAKING IN WATER QUALITY CONTROL*

By

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Details of illustrations in this document may be better studied on microfiche.

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ABSTRACT

A method of utilizing existing water quality and hydraulic computer models to arrive at least cost solutions of water quality problems is presented. The alternatives considered are reduction of waste loads by treatment, industrial process modification, and construction of reservoirs for low flow augmentation. The application of the methods to water resources problems in the James River Basin is discussed. The least cost solution of the water quality control problem is presented along with a subjective analysis of various alternatives.

Key Words: Water Pollution, Mathematical Models, Low-flow Augmentation, Systems Analysis, Optimization Methodology, Decision-making, James River Basin

INTRODUCTION

The Chesapeake Field Station* of the Federal Water Pollution Control Administration has been participating in water quality control management and planning studies for the past several years. Since its inception, the Field Station has attempted to make maximum use of the newly developing tools of systems analysis and operations research in its efforts. It quickly found that in this area there is a rapidly growing wealth of theory, and that much of the theory has been translated to mathematical forms and programmed for rapid solution on high speed digital computers. However, it was also found that the application of these new tools to the solution of real life problems is not widely practiced. It is their application to an actual water quality problem that is the subject of this paper.

The primary objective of the study being presented was to determine the optimum blend of waste treatment and storage (for low flow augmentation) necessary to meet a specified dissolved oxygen (DO) standard in the James River Basin.

* During the period in which the study reported on here was made, both authors were employed at the Chesapeake Field Station.

THEORETICAL CONSIDERATIONS

Before discussing the case study, a description of the computer models used and the analytical concepts with which the study was approached will be presented.

The Computer Models Utilized -

The theory behind the actual computer programs and models utilized will be only dealt with briefly here since the objective of this paper is not a discussion of modeling techniques but to show how existing models and programs can be used to aid in the decision-making process. The three computer programs described below which have been utilized in the study are well documented elsewhere.

1. Oxygen Sag Model (1)

This program uses the basic Streeter-Phelps (2) formulation of the oxygen sag curve, a function describing the inter-action of the fundamental biological and physical processes that simultaneously remove and regenerate dissolved oxygen in a free flowing stream. The program offers adaptations to the basic formula to enable the inclusion of the effects of benthic organisms and photosynthesis on the oxygen balance. As programmed, the model is capable of computing the flow required to maintain a stated DO objective in a river system receiving multiple point source waste loads. It is important to realize that, as used in these studies, this computed flow requirement is totally unrelated to the actual hydrology of the system. It is, in fact, a theoretical flow required to meet the quality constraint one hundred percent of the time.

2. Segmented Esutary Model -

This program was designed to simulate the oxygen relationships in a tidal river or estuary. The basic principles of the model utilized were developed by Thomann.[3] The model incorporates a term to account for longitudinal diffusion caused by tidal action into the Streeter-Phelps equation and employs an incremental or segmented approach to the problem rather than a continuous function solution. Both the steady state [4] and time dependent [5] versions of this model have been programmed. The steady state version was used in the present study to compute the flow requirements necessary to meet the quality objective in the tidal segment of the river. In essence, this model serves the same purpose in the tidal river as the Oxygen Sag Model does in the free flowing portions.

3. Hydrologic Simulation Model -

This program[6] simulates the hydrologic nature of a river system. Stream flows (either historic or synthetic) are routed through the river system and compared to required target flows (computed outside of the model) at various locations in the basin and the pertinent statistics regarding failures computed. Reservoirs can be introduced into the system and their effect on the statistics noted. In short, this model provides the means of determining the reliability of the hydrology in the basin to meet the required flows and the effect variation in the amount of storage provided has on this reliability.

Analytical Concepts -

The following theoretical discussion presents a general outline of the computational procedures with which the study was approached.

These procedures utilize the three river system models described above to arrive at the least cost combination of treatment and storage that

will satisfy DO quality objectives at various probabilities of failure (reliability).

In this theoretical discussion, only the following two policy constraints were assumed.

1. Treatment and storage will be given equal consideration. That is, no uniform treatment policy will be adopted and computations will begin with a system having no treatment and no storage for water quality control. The base could just as well be some level of treatment and some existing reservoir configuration.

2. A water quality (DO) standard has been established. (The procedure could be applied to various other standards to show the cost of attaining these standards).

In addition to these policy constraints, it was assumed that the following as inputs to the river system models were available.

1. Waste loadings (lb/day 5-day BOD)
2. Parameters defining the self-purification of the river (K_1, K_2, K_3, etc.)
3. Layout of system geometry
4. Historic river discharge records
5. Statistical analysis of water temperature

Given a particular river system, the flow required to assimilate a specific waste load and maintain a specified water quality is primarily a function of temperature. By selecting design temperatures for seasons corresponding to the seasons for which the analyses are to be made (e.g., months) a minimum flow requirement that will meet the quality constraint can be computed for each month using either the Oxygen Sag or Segmented Estuary Model. These seasonal flow requirements can be arranged to form a matrix, the columns

of which represent the months and the rows of which represent the various waste loading levels (either changedby improved waste treatment or, in the case of industrial waste, a percent reduction in loadings due to process changes).

Waste Loading Level	Matrix A Flow Required to Meet Quality Objective Months Jan.	Feb.	Mar.	Apr.	May	. . . . .	Dec.
1	Q_{11}	Q_{12}	.	.	.	.. ..	.
2	Q_{21}	Q_{22}	.	.	.	.. ..	
3	Q_{31}						
4	.						
5	.						
.	.						
.	.						
1	.						
n	.						

For example, in Matrix A shown above, row one might represent the flow required with no treatment and column one represents the flow required in January.

The row matrices of Matrix A can be introduced into the Hydrologic Simulation model described previously to compute the failure probability for each element of the row matrices as well as for the row matrix itself. This program is capable of handling a large river system with various waste loadings and thus minimizing the quantity of data coding and the number of computer runs

required. The probability of failure for each location in the river system is computed by the program simply as the number of deficient seasons divided by the total number of seasons on the historic or synthetic record. The vector of probabilities formed by consideration of the different waste loading levels is essentially a conversion of the elements in Matrix A to a vector, the components of which are failure probabilities for the system at various waste loading levels.

Similarly, by step increasing the potential storage capacity of a reservoir placed in the system and rerouting the dilution flow requirements through the hydrologic model, a number of such probability vectors can be computed.

A family of curves as shown for Figure 1 can be drawn from the data thus generated.

A new matrix can then be extracted from these curves. The rows of this matrix correspond to allowable loads and the columns refer to selected risk factors (probabilities).

Matrix B

Allowable Loading Levels	Quantity of Storage, cfs-months Risk Factor, 1 month per x years				
	1/5	1/10	1/20	1/30	1/50
T_1	S_{11}				
T_2					
T_3					
T_4					
T_5					
n					S_{ij}

Given cost curves for both storage and treatment, the elements of the second matrix can be mapped into cost. By adding the cost for treatment to each of the corresponding elements in the cost matrix, a matrix of total system cost is formed. The position of the minimum cost in each column will identify the least cost treatment and storage combination to meet the desired standard at that particular failure rate. The costs for each failure rate can then be ranked, listing the corresponding combination of treatment and storage.

III. A CASE STUDY - THE JAMES RIVER BASIN

The case study presented herein is a preliminary evaluation of the potential of the water resources in the James River Basin for meeting present and future water needs. The study was performed by the authors while employed by the Federal Water Pollution Control Administration as part of the Water Resources Study for Appalachia authorized by Congress in the Appalachian Regional Development Act of 1965. The Federal Water Pollution Control Administration, cooperatively with the U.S. Corps of Engineers, Virginia State Department of Water Resources, Virginia State Water Control Board, and other Federal and State agencies, is currently evaluating present and future water resource needs of the James River Basin toward formulating a comprehensive program to enhance and preserve the beneficial water uses in the Basin. While the case study presented in this paper was based on information existing at the time of the study, the findings are not final and should not be interpreted as representing the Basin program to be formulated.

General Basin Description -

The James River Basin is narrow and irregular with headwaters in the Allegheny Mountains at the West Virginia state line. The river flows generally southeasterly 340 miles through four physiographic regions: the Valley and Ridge, the Blue Ridge, the Piedmont, and the Coastal Plain (Figure 2). The total area drained is 10,060 square miles, of which 80 are in West Virginia at the edge of the Appalachian region. There is a total fall of 988 feet from the headwaters to the "Fall Line" separating the Piedmont and Coastal Plain at Richmond. From this point, the James is an estuary that joins the Chesapeake Bay at Hampton Roads.

Here we will focus our attention on that portion of the main stem of Craig Creek from a proposed dam site (Hipes) to its confluence with the James River, that portion of the Jackson River extending from the Gathright Dam site to its confluence with the James River and the main stream of the James River from its confluence with the Jackson River up to and including the James estuary immediately below the City of Richmond.

Gathright is a multipurpose reservoir under construction on the Jackson River approximately 19 miles upstream from Covington, Virginia, which will provide 60,700 acre feet of storage for flood control, recreation and water quality control.

The Hipes site on Craig Creek has been selected by the Corps of Engineers as the most feasible site for reservoir storage in the basin both from an economic and design standpoint. In this study, our analysis was limited to increased storage at the Hipes site for water quality control. Reservoir cost data for this site were provided by the U.S. Corps of Engineers.

Water Quality Control Flow Requirements

Dissolved oxygen is the water quality indicator upon which the design of control measures was based in this study. A minimum monthly average DO of 5.0 mg/l was the planning objective.

There are presently two areas in the James River Basin, Lynchburg and Richmond, within the sphere of influence of the Hipes reservoir site that experience recurrent DO problems. Figure 3 indicates the effect of wastes discharged in these two vicinities on the DO of the river. Economic and engineering projections of industrial and municipal waste loads to the James River showed that, although the provision of biological treatment would improve the existing conditions, the projected growth and resulting waste loads in these areas require that control methods in addition to conventional waste treatment be considered. A detailed description of the projections and resulting loading has been given elsewhere. (7)

To form a basis on which to plan for possible water quality control needs, the 50-year period ending in the year 2020 has been divided into two 25-year treatment plant design periods. The required monthly river discharge in the critical sections below Lynchburg and Richmond under present conditions and at the end of each of the 25-year periods was calculated using the water quality models previously discussed. The results are shown in Tables 1 and 2 for three levels of waste treatment. The waste treatment levels considered are defined as follows:

Level 1 - Conventional secondary treatment operated at an efficienty of 85 percent removal of 5-day BOD. The provision of this level or its equivalent in BOD reduction is considered to be prerequisite to low flow augmentation.

TABLE 1

WATER QUALITY CONTROL FLOW REQUIREMENT TO MAINTAIN 5.0 mg/l
AVERAGE DO OBJECTIVE IN THE JAMES RIVER AT LYNCHBURG

Design Period		Treatment Level(1)	Annual Cost(2) 1000's Dollars		Flow Requirements in cfs												Reliability Factor(3)
From	To		Initial	Final	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Present	N.A.	I	N.A.	N.A.	220	240	290	420	460	690	690	690	580	450	300	250	99.5
		II	N.A.	N.A.	200	220	260	380	410	640	640	640	510	400	270	230	
		III	N.A.	N.A.	180	200	230	340	370	570	570	570	470	360	240	210	
1970	1995	I	base	base	300	300	370	550	620	900	900	900	750	580	380	320	96.6
		II	175	205	240	270	320	480	540	780	780	780	670	500	330	280	98.6
		III	745	855	210	230	280	400	450	670	670	670	570	420	290	240	99.8
1995	2020	I	base	base	320	380	450	650	720	1100	1100	1100	900	700	470	400	91.9
		II	210	240	280	320	390	560	610	900	900	900	760	600	400	340	94.4
		III	1055	1180	240	270	310	450	490	700	700	700	600	470	330	280	99.5
Mean Monthly Flow(4)					4749	5822	7280	6149	4048	2437	1608	1959	1437	1919	2242	3445	
Minimum Monthly Flow(4)					631	690	2741	1798	1190	910	503	458	421	431	511	729	
Mean Monthly Maximum Temperature (°C)					9	11	15	23	25	32	32	32	29	24	16	12	

(1) Treatment Levels I, II, and III represent 85, 90, and 95 per cent BOD removal respectively.

(2) Total cost including operation, maintenance, and amortization of capital at three and one-fourth per cent for 25 years. (ENR = 1090).

(3) Per cent of months out of 37 years of historic record that flow requirements were met. Gathright assumed operating for maximum need at Covington.

(4) From USGS gage on the James River near Holcombs Rock, Virginia.

TABLE 2

WATER QUALITY CONTROL FLOW REQUIREMENT TO MAINTAIN 5.0 mg/l
AVERAGE DO OBJECTIVE IN THE JAMES RIVER AT RICHMOND

Design Period		Treatment Level(2)	Annual Cost(3) 1000's Dollars		Flow Requirements in cfs												Reliability Factor(4)
From	To		Initial	Final	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
Present	N.A.	I	N.A.	N.A.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	100.0
		II	N.A.	N.A.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	100.0
		III	N.A.	N.A.	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	100.0
1970	1995	I	base	base	300	300	300	650	1000	1450	1550	1550	1350	900	300	300	96.2
		II	90	160	(1)	(1)	(1)	(1)	300	650	800	800	550	(1)	(1)	(1)	100.0
		III	930	1,320	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	100.0
1995	2020	I	base	base	400	400	500	950	1250	1650	1800	1800	1550	1150	600	400	94.8
		II	170	210	(1)	(1)	(1)	(1)	500	900	1050	1050	800	300	(1)	(1)	99.5
		III	1,530	1,810	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(1)	100.0
Mean Monthly Flow(5)					10021	11662	13569	12066	7392	5238	3605	4520	3697	4138	4471	7387	
Minimum Monthly Flow(5)					1656	4103	6396	3776	3013	1684	1306	858	802	700	1113	1681	
Mean Monthly Maximum Temperature (°C)					8	9	13	20	24	29	30	30	28	23	16	12	

(1) Water quality control flow requirements are less than minimum daily flow recorded.

(2) Treatment Levels I, II, and III represent 85, 90, and 95 per cent BOD removal, respectively.

(3) Total annual cost including operation, maintenance, and amortization of capital at three and one-fourth per cent for 25 years (ENR = 1090).

(4) Per cent of months out of 37 years of historic record that flow requirements were met. Gathright assumed operating for maximum need at Covington.

(5) From USGS gage on the James River at Richmond (adjusted to include Kanawha Canal diversion).

Level II - Ninety percent removal of the 5-day BOD. This level will result in a further one-third reduction in treated waste load discharges and can be achieved by design modifications of conventional biological waste treatment plants.

Level III - Ninety-five percent removal of the 5-day BOD. This level of treatment can be achieved by conventional biological treatment in combination with oxidation ponds, chemical precipitation or some other method of advanced waste treatment.

The cost estimates shown in the tables for each treatment level represent the annual expenditures at the beginning and end of the design period to provide either Level II or III, over and above the annual cost of Level 1. Level 1 is considered to be a minimum level when considering low flow augmentation as an alternative to treatment. Costs were estimated from municipal treatment plant cost data compiled and analyzed by Frankel (8) and from cost data made available by industries in the Basin. The costs shown include total annual cost of operation, maintenance and amortization of capital at 3-1/8 percent over 25 years. The interest rate chosen is the same as that used by the U.S. Corps of Engineers, Norfolk District, for reservoir construction. Uniformity of interest rate was considered to be desirable for benefit evaluation.

The stream flow requirements necessary to assimilate the corresponding waste loads without depleting the average DO below 5.0 mg/l were computed by means of the digital computer simulation models discussed previously. For planning purposes, the mean monthly maximum water temperatures were used.

The probability at which the natural or regulated stream flows can be expected to meet the required stream flows is a measure of the reliability of the system. This probability is computed as the number of successful events divided by the total number of events considered. As applied to the monthly stream flow requirements, an event consists of a routing of an historical monthly average stream flow through the river system and comparing it to the corresponding monthly flow requirement. In this study we used the term "reliability factor", defined as the probability, as discussed above, multimplied by 100.

$$\text{Reliability Factor} = 100 \times \frac{\text{Number of Successful Months}}{\text{Total Number of Months in Historic Record}}$$

Thirty-seven years of historical stream flow data have been routed through the system and reliability factors computed for each of the flow requirement vectors shown in Tables 1 and 2. For planning purposes, a reliability factor of 99.6 has been chosen. On the basis of monthly flows, this represents one unsuccessful month out of 20 years. Hydrologic analysis of the historical records showed that this reliability is approximately equivalent to the 7-day, 10-year return frequency drought flow specified in the State of Virginia proposed water quality standards. It was assumed that Gathright Reservoir will be operational by the early 1970's. Hydrological routings, therefore, have been made with Gathright operating for maximum water quality control benefit. Table 3 shows the target flows assumed for operation of Gathright Reservoir.

The hydrological analysis has been facilitated by the application of a computer program developed by Fiering and Pisano discussed earlier. Thirty-seven years of historic stream flow records were available for this study area. The period of record used was from October 1927 through September 1964. Figure 4 shows the system geometry as assumed for this study.

TABLE 3

GATHRIGHT RESERVOIR RELEASE RULES

Operation for Year 2020 Water Supply and Water Quality
Control Flow Requirements at Covington

Design Year	Treatment Level	Flow Past Covington in cfs											
		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2020	I	175	170	180	190	210	365	365	365	245	210	185	175
Design Temperature		9	8	12	20	21	26(1)	26(1)	26(1)	24	21	13	10

(1) Based upon controlled temperature release from Gathright.

Table 4 is a list of the U.S. Geological Survey stream gaging stations used and Table 5 lists the pertinent geographical locations of the system. Gage No. 6 was created for this study by adding the flows from the gage on Dunlap Creek with those at Falling Spring on the Jackson River. The resulting record was considered to be a reasonably good approximation of the flows that occur at Covington, the point of need for Gathright water quality control storage.

TABLE 4

STREAM GAGES USED IN HYDROLOGIC ANALYSIS OF JAMES RIVER BASIN

Gage No.	Description	Drainage Area (square miles)
1	Jackson River at Falling Spring	409
2	James River at Lick Run	1369
3	James River at Buchanan	2084
4	James River at Holcombs Rock	3250
5	James River at Bent Creek	3671
6	Dunlap Creek plus Falling Spring gage	575
7	James River at Casterville	6242
8	Cowpasture River near Clifton Forge	456
9	Craig Creek at Parr	331
10	Dunlap Creek near Covington	166
11	Maury River at Buena Vista	649

TABLE 5

DESCRIPTION OF COORDINATES FOR HYDROLOGIC ANALYSIS

Location	River Mile	Drainage Area (square miles)	Reference Gage(1)	Gage Adjustment Factors
Jackson River Gathright dam site	43.5	344	1	0.8411
Jackson River City of Covington	23.5	610	6	1.0000
Craig Creek Hipes dam site	14.8	327	9	1.0000
James River City of Lynchburg	250.0	3200	4	1.0000
James River City of Richmond	100.0	6760	7	1.0816

(1) See Table 4 for gage description.

ALTERNATIVE METHODS OF MEETING WATER QUALITY CONTROL OBJECTIVES

Probably, the most widespread methods for meeting specific water quality objectives with respect to dissolved oxygen are: (1) reduction of BOD waste loads by treatment at the source; and (2) regulation of stream flow by providing reservoir storage. Preliminary studies indicated that other methods such as instream aeration, waste storage, and piping of waste to bodies of water having a greater assimilation capacity did not appear to be feasible devices for pollution control in the Lynchburg and Richmond areas.

The analysis was, therefore, limited to an examination of storage and treatment either separately or in combination, as a means of meeting the water quality control needs of the portion of the James River described. From a cost evaluation of many possible combinations of storage and treatment, a single least cost alternative can be selected. Table 6 defines the pollution control alternatives that have been considered for this report. The treatment combinations considered include all possible combinations of the three treatment levels at Lynchburg and Richmond. The amount of reservoir storage in Hipes that is required to meet the average DO objective of 5.0 mg/l for each of the treatment combinations has been computed as previously discussed.

Figures 5 and 6 exemplify the method of handling the output from the program to determine storage requirements. By routing the historic flows through the system with various sizes of storage in Hipes and plotting the computed failure probabilities versus storage, the storage required to meet the desired reliability can be picked from the curve. These curves were

TABLE . 6

STORAGE REQUIREMENTS CORRESPONDING TO ALL POSSIBLE COMBINATIONS OF TREATMENT LEVELS AT LYNCHBURG AND RICHMOND

Treatment Level		Required Storage in Pipes (acre-feet)
Lynchburg	Richmond	
Design Period 1970 - 1995		
I	I	75,000
I	II	27,000
I	III	27,000
II	I	75,000
II	II	0
II	III	0
III	I	75,000
III	II	0
III	III	0
Design Period 1995 - 2020		
I	I	128,000
I	II	62,000
I	III	62,000
II	I	[illegible]
II	II	[illegible]
II	III	27,000
III	I	128,000
III	II	0
III	III	0

developed for all of the alternatives considered. Only the curves for the selected alternate are shown as an example.

In the historic record of thirty-seven years, two months' failure would exceed the objective reliability factor of one month in twenty years. However, based on observations of the records from gages for which a longer historic trace is available, it was concluded that, for the limited record used, two failures (reliability factor = 99.5) would be accepted for design purposes. Future studies, perhaps utilizing synthetically generated flow records, are recommended to support this judgment decision.

The total pollution control alternative, therefore, consists of a specific treatment level at Lynchburg and at Richmond in conjunction with a specific storage volume at the Hipes site for stream flow regulation.

Since the benefit of storage for water quality control is generally evaluated on the basis of a fifty year period, it is necessary to select the long-range alternative first and then designate the minimum required short-term action plan, taking fullest advantage of the storage computed to meet the long-range needs. By so doing, the alternatives listed in Table 6 can be reduced to the six logical fifty-year pollution control alternatives shown with their respective costs in Table 8.

Pollution control by treatment is generally designed for a twenty to twenty-five year period of capital amortization. For comparative purposes, the alternatives and associated costs for a twenty-five year plan are given in Table 7.

DISCUSSION OF ALTERNATIVES

Alternatives 1 through 9 (see Table &) are designed to meet the stream DO objective through the year 1995. These alternatives are included

TABLE 7

COST COMPARISON FOR POLLUTION CONTROL ALTERNATIVES

Twenty-Five Year Plan

(1970 - 1995)

Alternative Number	Description of Alternative: Treatment Level Lynchburg	Richmond	Storage In Hipes (acre-ft.)	Cost of Alternative Present Worth 1970: Treatment	Storage	Total
				(millions of dollars)		
1	I	I	75,000	0.00	9.87	9.87
2	I	II	27,000	2.15	6.10	8.25
3	II	II	0	5.41	0.00	5.41
4	II	I	75,000	3.52	9.87	13.39
5	III	II	0	15.88	0.00	15.88
6	III	I	75,000	13.74	9.87	23.61
7	I	III	27,000	19.32	6.10	25.42
8	II	III	0	22.58	0.00	22.58
9	III	III	0	33.06	0.00	33.06

TABLE 8

COST COMPARISON FOR POLLUTION CONTROL ALTERNATIVES

Fifty-Year Plan
(1970 - 2020)

Alternative Number	Description of Alternative: 1970 - 1995 Treatment Level		1995 - 2020 Treatment Level		Storage in Hipes (acre-ft)	Cost of Alternative Present Worth, 1970 (millions of dollars)		
	Lynchburg	Richmond	Lynchburg	Richmond		Treatment	Storage	Total
10	I	I	I	I	123,000	0.00	11.21	11.21
11	I	I	I	II	75,000	1.51	9.87	11.38
12	I	II	II	II	27,000	5.45	6.10	11.55
13	II	II	III	II	0	15.67	0.00	15.67
14	III	II	III	II	0	26.15	0.00	26.15
15	III	III	III	III	0	55.26	0.00	55.26

for comparison purposes. It is presently common practice to assume a fifty-year design period for storage for water quality control. These shorter range plans are presented because there are definite advantages to short-range planning in this day of rapid technological change. Advances in waste treatment technology and industrial production purposes could drastically alter the water requirements and waste production of the future.

Alternatives 10 through 15 are six feasible long-range (fifty years) water quality control plans from which the need for and value of storage in a multi-purpose reservoir on Craig Creek can be evaluated. Table 9 provides a subjective evaluation of these six alternatives. Although the rating of secondary effects is admittedly a somewhat arbitrary approach to making a non-monetary comparison of the different plans, it does present a method of acknowledging and considering effects that might not otherwise be included in the evaluation.

The following is a short description of each of the six fifty-year plans. The particularly significant secondary effects of each are also briefly commented upon.

Alternative 10 is what can be considered the minimum acceptable treatment level in combination with 128,000 acre-feet of storage for water quality control at the Hipes site. This is both the least costly alternative and the one requiring the most storage. Significant secondary benefits are increased low flows in the Fall Zone at Richmond and maximum performance stability. Since this alternative requires the minimum treatment and maximum storage, it is the least consistent with the goal of complete elimination of point-source pollution.

Alternative 11, the second least costly alternative, is judged insignificantly more expensive than Alternative 10 and the storage requirement

TABLE 9

SUBJECTIVE EVALUATION OF POLLUTION CONTROL ALTERNATIVES

Secondary Effects[1]	Pollution Control Alternative Rating[2] 10	11	12	13	14	15
a. Nutrient removal above Richmond for water supply and below Richmond for estuary eutrophication	2	2	2	3	3	4
b. Color removal--associated with paper industry, particularly below Lynchburg	2	2	2	3	4	4
c. Recreation--effects on river	2	2	2	2	1	1
d. Fishery--effects on river	?	?	?	?	?	?
e. Ability to take advantage of technological advances	-	1	2	3	4	4
f. Variability in performance	4	3	3	2	1	1
g. Aesthetic improvement--flow in Fall Zone at Richmond	4	4	3	2	1	1
h. Consistent with goals of complete elimination of point source pollution	-	2	2	3	3	4
TOTAL	12	16	16	18	17	19

(1) Effects incidental to principal objective of maintaining 5.0 mg/1 DO.

(2) Definition of Ratings
- Negative effect
1 No effect
2 Slight positive effect
3 Significant effect
4 Very significant effect
? Unknown effect

is reduced by about forty percent to 75,000 acre-feet. This reduction in storage is the result of the application of the second treatment level at Richmond beginning in 1995 and through the year 2020. The secondary benefits of this alternative are essentially the same as the first, with an additional slight positive effect in the area of eliminating point-source pollution.

Alternative 12 calls for the second treatment level at Richmond for the 1970 through 1995 design period and at both Richmond and Lynchburg for the 1995 to 2020 design period. The alternative requires 27,000 acre-feet of storage at Hipes and has a total cost that is again only insignificantly more expensive than the least costly plan. In the subjective evaluation, losses with respect to esthetic improvement for the Fall Zone at Richmond are balanced by gains in the ability to take advantage of technological advances.

Alternative 13 is almost one and one-half times as costly as the least cost alternate. The sharp rise in cost is due to the requirement of treatment level III which involves chemical precipitation and its correspondingly high operating costs. No storage above that provided by the Gathright Project is required. The 95 percent treatment level with chemical precipitation at Lynchburg would have the secondary benefit of significant color and nutrient reduction.

Alternative 14 requires the third level of treatment at Lynchburg for both design periods and the second level of treatment at Richmond for both design periods. No flow regulation other than that provided by Gathright for the Covington area is needed. The cost is substantially more than Alternative 13. The secondary benefits are somewhat higher in the nutrient and color removal categories but lower in performance variability and dependable flow in the Fall Zone.

Alternative 15 is five times as expensive as the least cost alternative due to the specified highest treatment level for all waste contributors in the Lynchburg and Richmond areas from 1970 through 2020. As in Alternatives 12 and 14, no storage is required. This alternative scores highest in scondary effects with very significant positive effects on nutrient and color removal, ability to take advantage of technological advances and consistency with the goal of complete elimination of point-source pollution.

SUMMARY AND CONCLUSIONS

In making this study, many conclusions were reached, some of which can be supported by the data presented while others are more subjective in nature. The more significant of these are:

1. In a given situation, low flow augmentation is still an economically feasible method of water quality control.
2. In using operations research and systems analysis principles in water quality control planning, translating theory to mathematical formulas and computer programming is only a small portion of the task facing engineers and scientists. Obtaining the data necessary to satisfy the input requirements of these models and evaluating and interpreting output in the light of political, economic and physical constraints, in fact, pose significant if not more difficult problems.
3. There is a need for greater emphasis by water quality control agencies on the establishment of water resource policy. Studies on the implication of various policy avenues by use of the case study technique are desirable.
4. From the experience encountered in the James River studies, the solution of the least cost alternative does not seem to lend itself, practically speaking, to a continuous solution. Cost curves for waste treatment are generally not continuous functions especially when complicated by existing treatment levels and policy constraints. The individual evaluation of discreet management plans not only is indicated to be a more practical approach but perhaps a more desirable and realistic one as well.

5. Mathematical models in water pollution control planning can provide a means whereby many system designs may be evaluated and thereby provide a more rational selection of the optimum design. It is the opinion of the writers, however, that the decision should not be based solely on the cost aspects of the total water resources picture. A subjective analysis of the project may show that the least cost solution may not be the optimum solution. In fact, there may be more than one "optimum solution" depending upon the vantage point of the decision maker.

6. Considerable field work followed by still more office work involving the use of large measures of engineering judgment were required to arrive at reasonable approximations of the system parameters and input data required for this study. For this reason, caution is advised in attempting to reduce all problems to a computerized systems analysis approach. For example, it probably would not be economically feasible to spend the money and manpower required to obtain models for very small, perhaps intermittent, creeks. In cases such as this a policy decision based on experience would appear to be more realistic.

7. Limited industrial waste reduction cost data present a major problem in making cost oriented evaluations such as this one although the help received from the industries in the study area indicated their willingness to provide such data as exist.

BIBLIOGRAPHY

1. Worley, J.L. - "A System Analysis Method For Water Quality Management by Flow Augmentation In A Complex River Basin" - Master's Thesis, Oregon University, 1963.

2. Streeter, H.W. and Phelps, E.B. - "A Study of the Pollution and Natural Purification Of The Ohio River - III. Factors Concerned In The Phenomena Of Oxidation And Re-Aeration" - Public Health Bulletin No. 146, U.S. Public Health Service, February, 1925.

3. Thomann, R.V. - "Mathematical Model For Dissolved Oxygen" - Journal Of The Sanitary Engineering Division, ASCE, Volume 89, No. SA5, Proc. Paper 3680, October, 1963, pp. 1-30.

4. Jeglic, John M. - "Mathematical Simulation of the Estuarine Behavior", Digital Computer Technology and Programming Analysis Memo No. 1030, Rev. A, General Electric Re-Entry Systems Department, Philadelphia, Pennsylvania, July, 1967.

5. Bunce, Ronald and Hetling, Leo J. - "The Steady State Segmented Estuary Model" - CB-SRBP Technical Paper No. 12, Federal Water Pollution Control Administration, Middle Atlantic Region, Charlottesville, Virginia, 1968.

6. Fiering, M.B. and Pisano, W.C. - "Synthesis and Simulation Package For Reservoir Planning" - prepared for Federal Water Pollution Control Administration, U.S. Department of Health, Education and Welfare, 1966.

BIBLIOGRAPHY (CONTINUED)

7. "Water Supply and Water Quality Control Study - Hipes Reservoir, Craig Creek, Virginia" - Prepared For The Corps of Engineers, U.S. Department of the Army, Army Engineers District, Norfolk, Virginia, February, 1968.

8. Frankel, R.J. - "Water Quality Management: An Engineering Economic Model For Domestic Waste Disposal", Ph.D. Dissertation, University of California, Berkeley, 1965.

PROBABILITY OF FAILURE VERSUS ALLOWABLE WASTE LOADING LEVEL

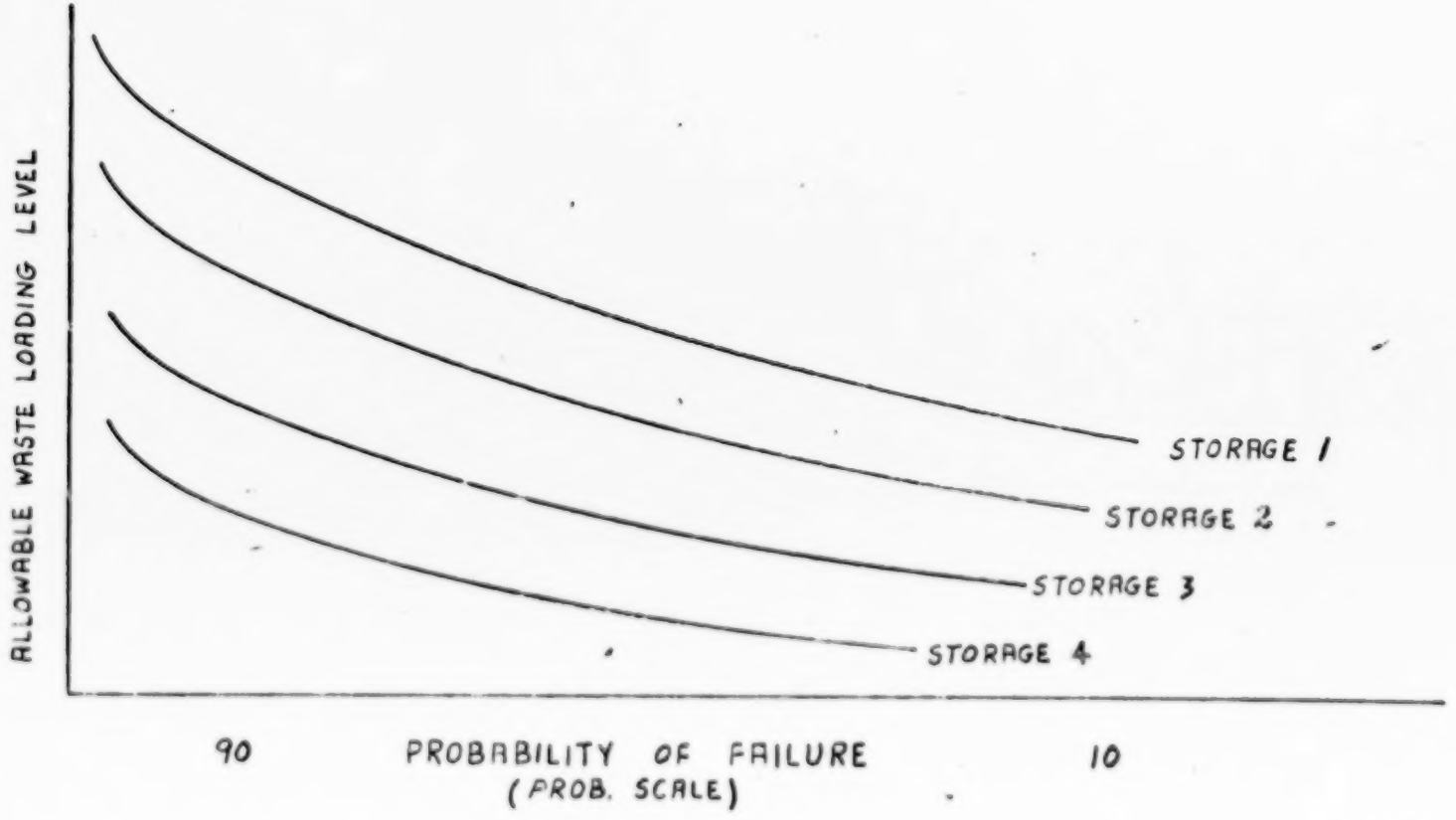
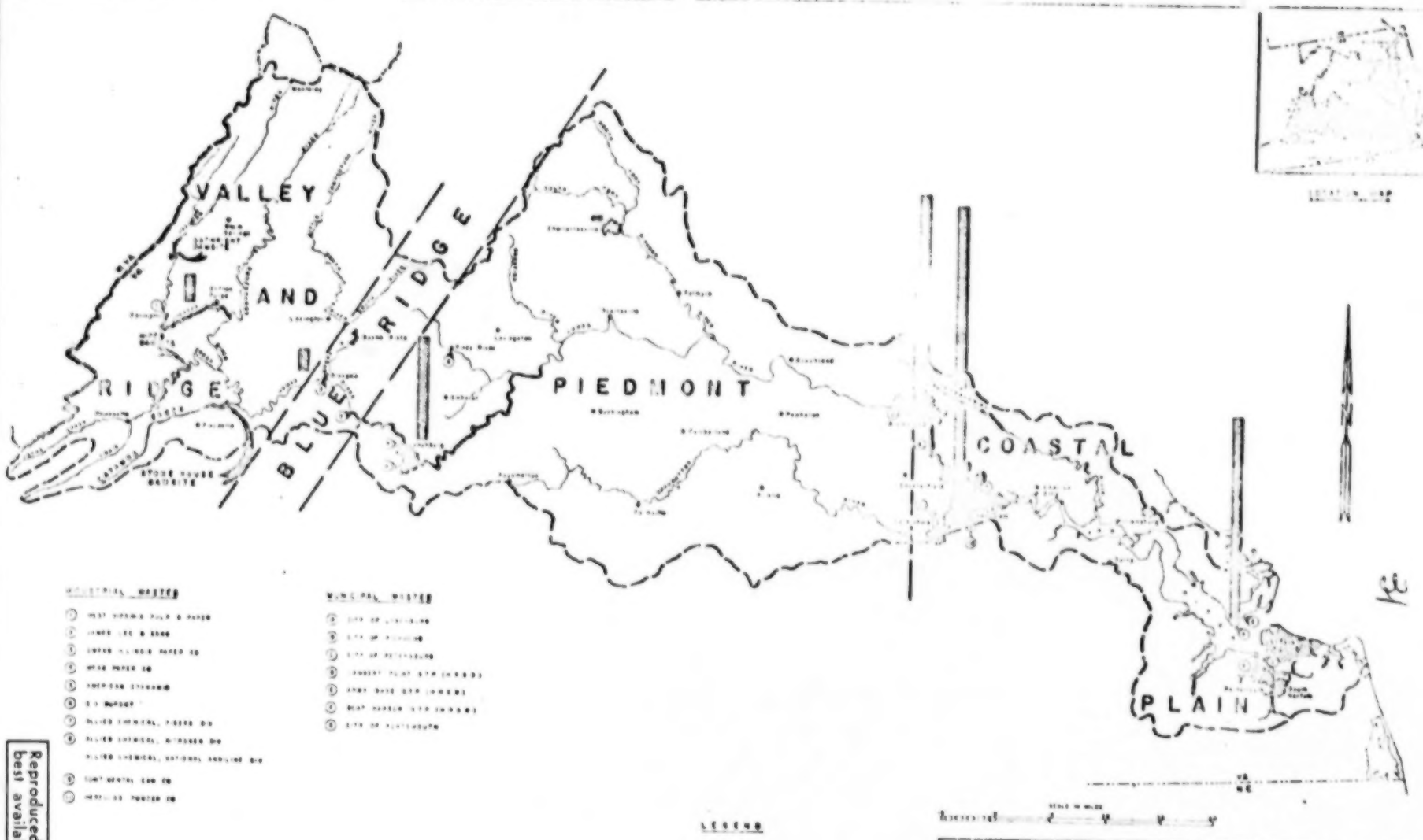


FIGURE 1



LEGEND

INDUSTRIAL WASTE P.E.

MUNICIPAL WASTE P.E.

SECTION OF STREAM EXPERIENCING RECURRENT CONDITIONS OF LOW DO AND/OR EUTROPHIC CONDITIONS

INDUSTRIAL WASTES

MUNICIPAL WASTES

PROPOSED RESERVOIRS

APPALACHIA PROGRAM
WATER SUPPLY & WATER QUALITY CONTROL STUDY

JAMES RIVER BASIN

PRINCIPLE MUNICIPAL & INDUSTRIAL WASTE SOURCES

U.S. DEPARTMENT OF THE INTERIOR
FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
MIDDLE ATLANTIC REGION CHARLOTTESVILLE, VA.

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JAMES RIVER

AVERAGE DISSOLVED OXYGEN FOR SAMPLING SURVEYS
CONDUCTED IN JULY AND/OR AUGUST, 1966

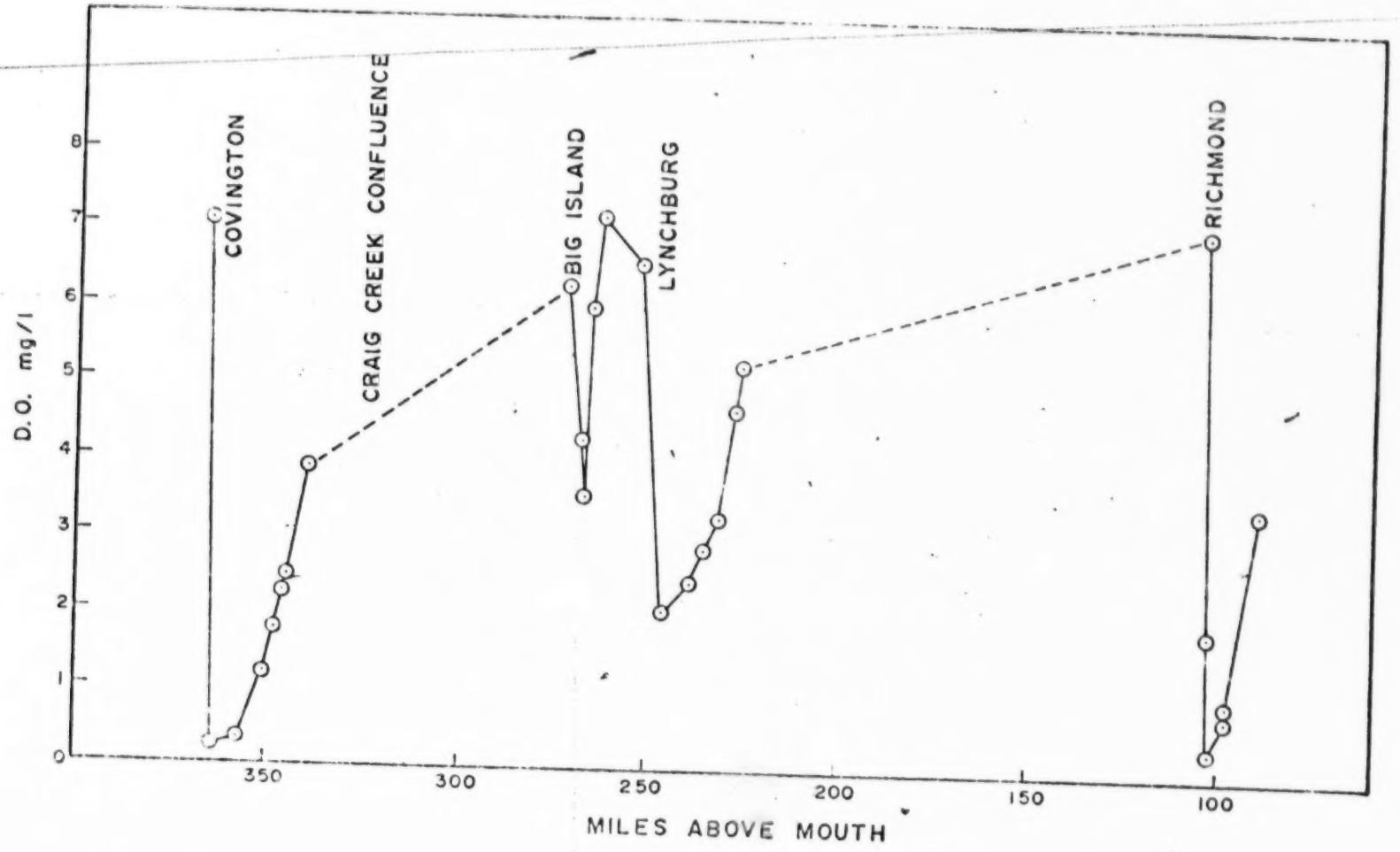

FIGURE 3

SIMPLIFIED SKETCH OF GEOMETRY
for
HYDROLOGIC ANALYSIS

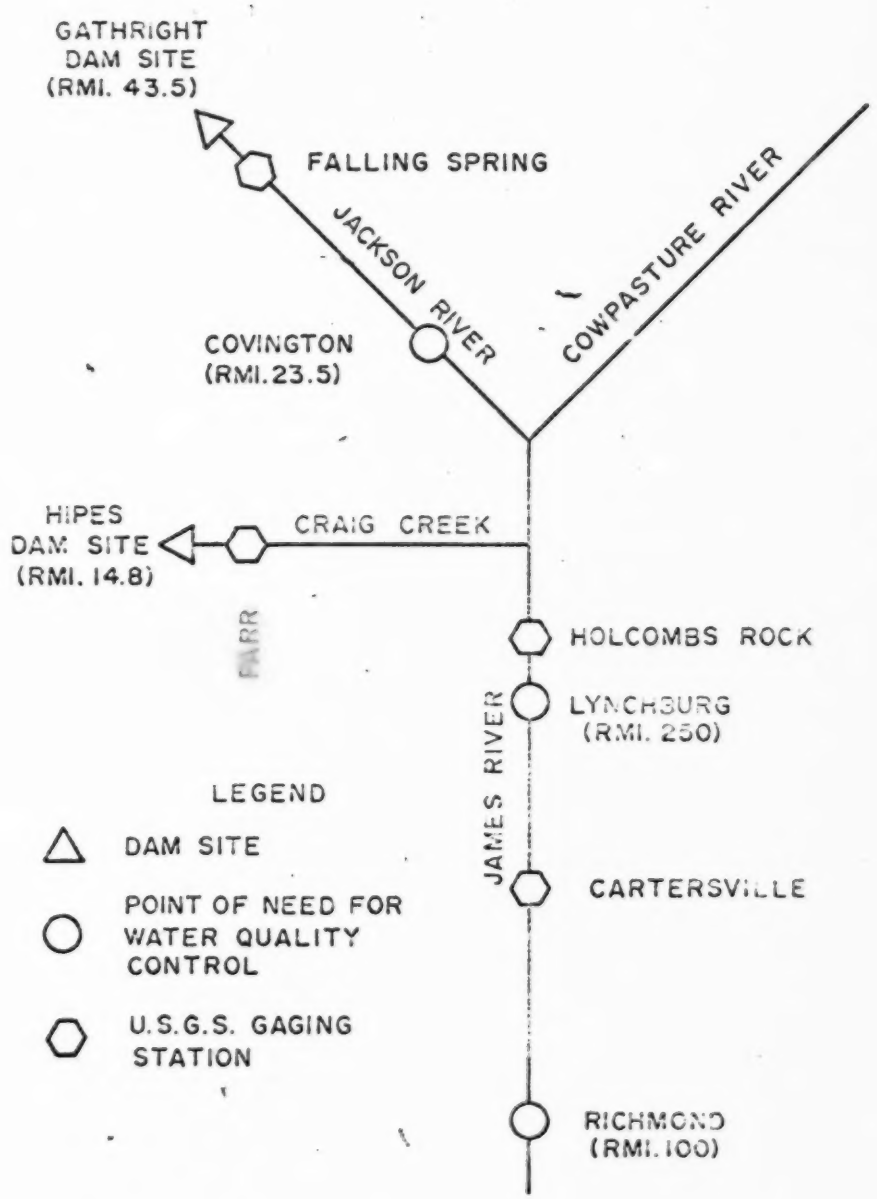

FIGURE 4

RELIABILITY OF WATER QUALITY CONTROL VS. STORAGE FOR LOW FLOW AUGMENTATION

(ALTERNATE NO. II AT 1995 CONDITIONS)

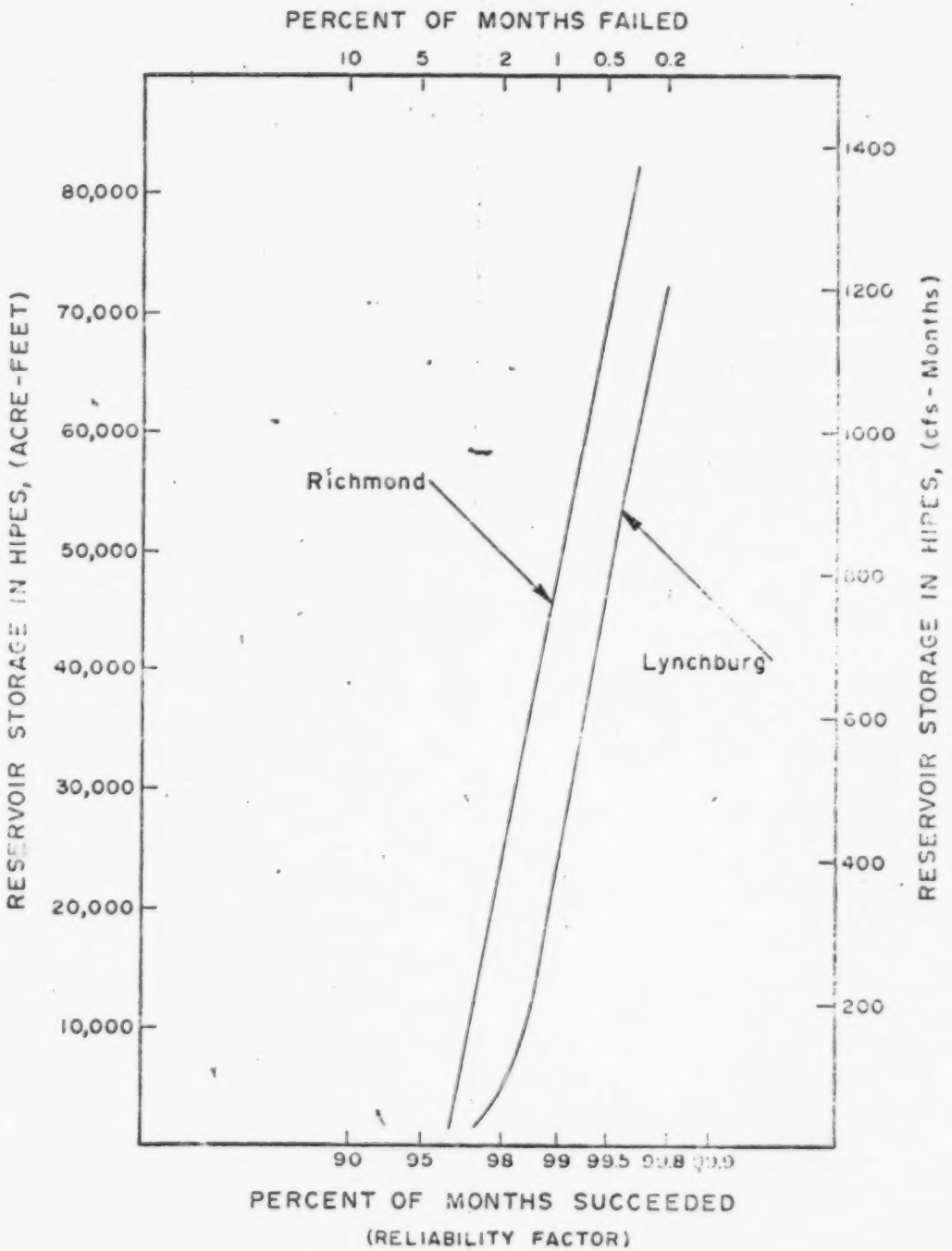

FIGURE 5

RELIABILITY OF WATER QUALITY CONTROL VS. STORAGE FOR LOW FLOW AUGMENTATION

(ALTERNATE NO. II AT 2020 CONDITIONS)

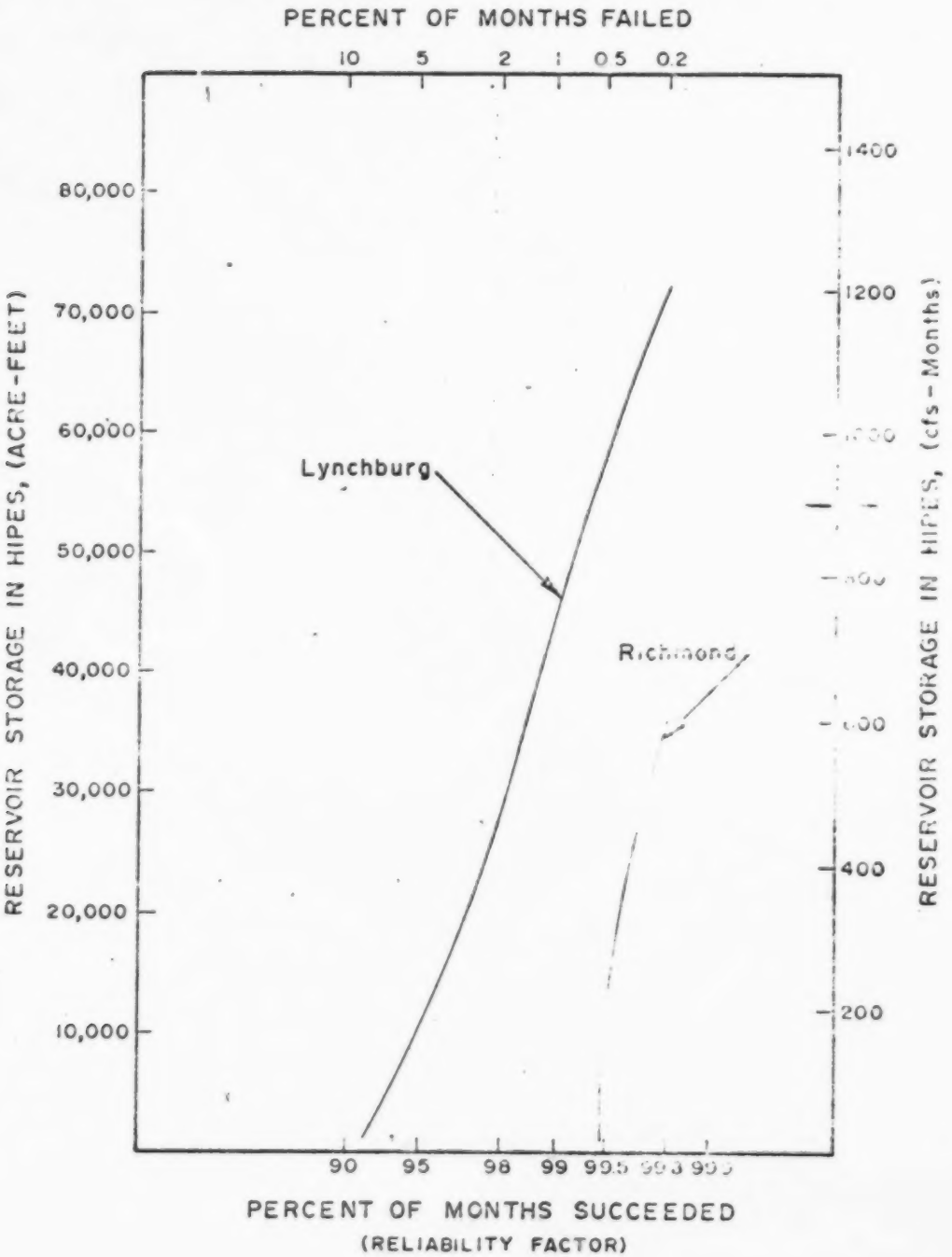

FIGURE 6